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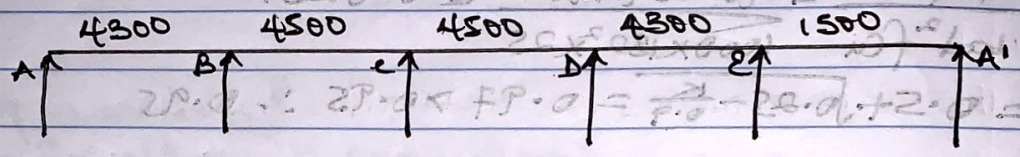
COURSE: STRUCTURAL DESIGN CVE308
MATRIC NO: 17/ENG03/1054
TOPIC: LMS ASSIGNMENT 2

1)




Assuming, thickness = 150mm
$f_{cu} = 25 N/mm^2$
$f_y = 410 N/mm^2$

**Slab loading**

Slab weight = $0.15 \times 24 = 3.6 KN/m^2$
Partition = $1.0 KN/m^2$
finishes = $1.2 KN/m^2$
Total G.K = $5.8 KN/m^2$

D.L = $1.4(5.8) + 1.6(3.0) = 13 KN/m^2$

**Beam loading**

Self weight of beam = $0.225 \times 0.6 \times 24 = 5.24 KN/m^2$
finishes = $1.2 KN/m^2$
Wall load = $3 \times 3.47 = 10.41 KN/m^2$
Total G.K = $14.85 KN/m^2$

D.L = $1.4(14.85) = 20.79 KN/m^2$

Slab load on beams in longer direction = $\frac{1}{2} w l_x \left(1 - \frac{1}{3K^2}\right)$

$K = \frac{l_y}{l_x}$

$\frac{4300}{4000} = 1.075$

$\frac{4500}{4000} = 1.125$

$\frac{1}{2} \times 13 \times 4.3 \left(1 - \frac{1}{3 \times (1.075)^2}\right) = 19.89 \text{ KN/m}^2$

$\frac{1}{2} \times 13 \times 4.5 \left(1 - \frac{1}{3 \times (1.125)^2}\right) = 21.55 \text{ KN/m}^2$

Slab load on beam in short direction = $\frac{1}{3} w l_x$

$= \frac{1}{3} \times 13 \times 1.5 = 6.5 \text{ KN/m}^2$

Total load = $19.89 + 20.79 = 40.68 \text{ KN/m}$

$= 21.55 + 20.79 = 42.34 \text{ KN/m}$

$= 6.5 + 20.79 = 27.29 \text{ KN/m}$

## Distribution Factor

$K_{AB} = 1$

$K_{BA} = \frac{\frac{1}{l_{BA}}}{\frac{1}{l_{BA}} + \frac{1}{l_{BC}}} = \frac{\frac{1}{4.3}}{\frac{1}{4.3} + \frac{1}{4.5}} = 0.51$

$K_{BC} = 1 - 0.51 = 0.49$

$K_{CB} = \frac{\frac{1}{4.5}}{\frac{1}{4.5} + \frac{1}{4.5}} = 0.5$

$K_{CD} = 1 - 0.5 = 0.5$

$K_{DC} = 0.49$

$K_{DE} = 0.51$

$K_{ED} = \frac{\frac{1}{4.3}}{\frac{1}{4.3} + \frac{1}{1.5}} = 0.26$

$K_{EA} = 1 - 0.26 = 0.74$

$K_{AE} = 1$

F.E.M

U.D.L = $\frac{wl^2}{12}$

1) $\frac{40.68 \times 4.3^2}{12} = 62.68 \text{ KN/m}$

2) $\frac{42.34 \times 4.5^2}{12} = 71.45 \text{ KN/m}$

3) $\frac{27.29 \times 1.5^2}{12} = 5.1 \text{ kNm}$

| | A | | B | | C | | D | | E | | A' | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | AB | BA | BC | CB | CD | DC | DE | ED | EA' | A'E | A' | |
| D.F | 0 | 1 | 0.51 | 0.49 | 0.5 | 0.5 | 0.49 | 0.51 | 0.26 | 0.74 | 1 | 0 |
| FEM | | -62.68 | 62.68 | -71.45 | 71.45 | -71.45 | 71.45 | -62.68 | 62.68 | -5.1 | 5.1 | |
| OBM | | -62.68 | -8.77 | 0 | | | 8.77 | | 57.58 | | -5.1 | |
| BM | | 62.68 | 8.77 | 0 | | | -8.77 | | -57.58 | | 5.1 | |
| DM | 0 | 62.68 | 4.47 | 4.30 | 0 | 0 | -4.30 | -4.47 | -14.99 | -42.61 | -5.1 | 0 |
| TM | | | 2.235 | 31.34 | 0 | 2.15 | -2.15 | 0 | -7.49 | -2.24 | -2.55 | -21.305 |
| OBM | | | 2.235 | 31.34 | 0 | | -7.49 | | -4.79 | | -21.305 | |
| BM | | | -2.235 | -31.34 | 0 | | 7.49 | | 4.79 | | 21.305 | |
| DM | 0 | -2.235 | -15.88 | -15.36 | 0 | 0 | 3.67 | 3.82 | 1.25 | 3.54 | 21.31 | 0 |
| TM | | | -7.99 | -1.12 | 0 | -7.68 | 1.84 | 0 | 0.63 | 1.91 | 10.66 | 7.77 |
| OBM | | | -7.99 | -1.12 | | -5.85 | | 0.63 | | 12.57 | | 1.77 |
| BM | | | 7.99 | 1.12 | | 5.85 | | -0.63 | | -12.57 | | -1.77 |
| DM | 0 | 7.99 | 0.57 | 0.55 | 2.43 | 2.43 | -0.31 | -0.31 | -3.27 | -9.29 | -1.77 | 0 |
| TM | | | 0.29 | 3.99 | 1.47 | 0.28 | -0.16 | 1.47 | -1.64 | -0.16 | -0.89 | -4.65 |
| OBM | | | 0.29 | 4.87 | 0.12 | | -0.17 | | -1.05 | | -4.65 | |
| BM | | | -0.29 | -4.87 | -0.12 | | 0.17 | | 1.05 | | 4.65 | |
| DM | 0 | -0.29 | -2.48 | -2.39 | -0.06 | -0.06 | 0.06 | 0.09 | 0.27 | 0.78 | 4.65 | 0 |
| Σ | | 0 | 22.88 | -82.88 | 69.06 | -69.06 | 72.06 | -72.06 | 45.47 | -45.46 | 0 | |

## Moments

$M_B = 0$ KNm

$M_C = 82.88$ KNm

$M_D = 69.06$ KNm

$M_E = 72.06$ KNm

$M_{A'} = 45.47$ KNm

$M_A = 0$ KNm

## Free moment

For U.D.L $= \frac{wl^2}{8}$

1) $\frac{40.68 \times 4.3^2}{8} = 94.02$ KNm

2) $\frac{42.34 \times 4.5^2}{8} = 107.17$ KNm

3) $\frac{27.29 \times 1.5^2}{8} = 8.24$ KNm

## Span moment

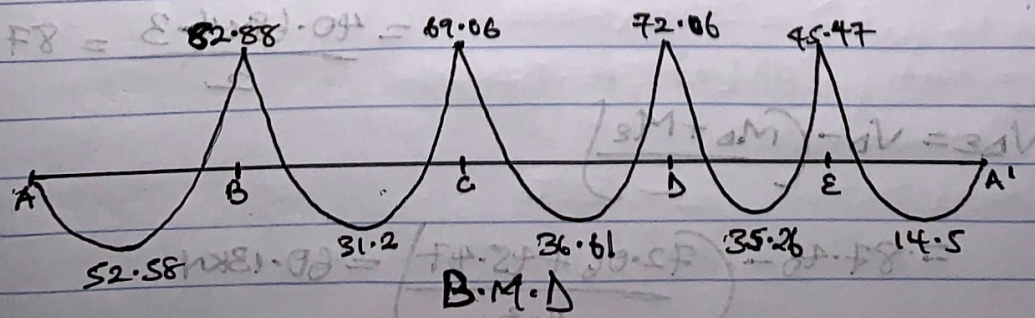
$M_{AB} = M^F - \left(\frac{M_A + M_B}{2}\right) = 94.02 - \left(\frac{0 + 82.88}{2}\right) = 52.58$ KNm

$M_{BC} = M^F - \left(\frac{M_B + M_C}{2}\right) = 107.17 - \left(\frac{82.88 + 69.06}{2}\right) = 31.2$ KNm

$M_{CD} = M^F - \left(\frac{M_C + M_D}{2}\right) = 107.17 - \left(\frac{69.06 + 72.06}{2}\right) = 36.61$ KNm

$M_{DE} = M^F - \left(\frac{M_D + M_E}{2}\right) = 94.02 - \left(\frac{72.06 + 45.47}{2}\right) = 35.26$ KNm

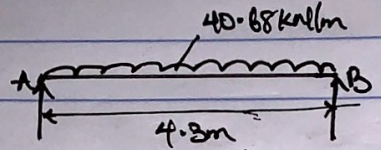
$M_{EA'} = M^F - \left(\frac{M_E + M_{A'}}{2}\right) = 8.24 - \left(\frac{45.47 + 0}{2}\right) = -14.5$ KNm




B.M.D

## Shear Force

for A




$$V_A = \frac{wL}{2} = V_B$$

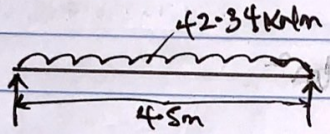
$$= 40.68 \times 4.3 = 87.462 KN$$

$$V_{AB} = V_B + \left(\frac{M_A - M_B}{L}\right) = 87.462 + \left(\frac{0 - 82.88}{4.3}\right) = 68.19$$

$$V_{BA} = wL - V_{AB}$$

$$= (40.68 \times 4.3) - 68.19 = 106.73 KN$$

for B




$$V_B = \frac{wL}{2} = V_C$$

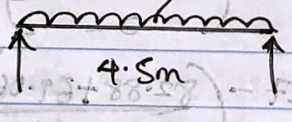
$$= \frac{42.34 \times 4.5}{2} = 95.27 KN$$

$$V_{BC} = V_B + \left(\frac{M_B + M_C}{L}\right) = 95.27 + \left(\frac{82.68 + 69.06}{2}\right) = 129.03 KN$$

$$V_{CB} = (42.34 \times 4.5) - 129.03$$

$$= 61.5 KN$$

for C




$$V_C = \frac{wL}{2} = V_D$$

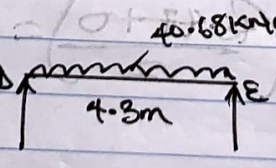
$$= \frac{42.34 \times 4.5}{2} = 95.27 KN$$

$$V_{CD} = 95.27 + \left(\frac{69.06 + 72.06}{4.5}\right) = 126.63 KN$$

$$V_{DC} = (42.34 \times 4.5) - 126.63$$

$$= 63.19 KN$$

for D




$$V_D = \frac{wL}{2} = V_E$$

$$= \frac{40.68 \times 4.3}{2} = 87.46 KN$$

$$V_{DE} = V_D - \left(\frac{M_D + M_E}{L}\right)$$

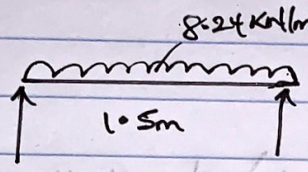
$$= 87.46 - \left(\frac{72.06 + 45.47}{4.3}\right) = 60.13 KN$$

$V_{ED} = wL = V_{DE}$

$= (40.68 \times 4.3) - 60.13$

$= 114.74 KN$

for E



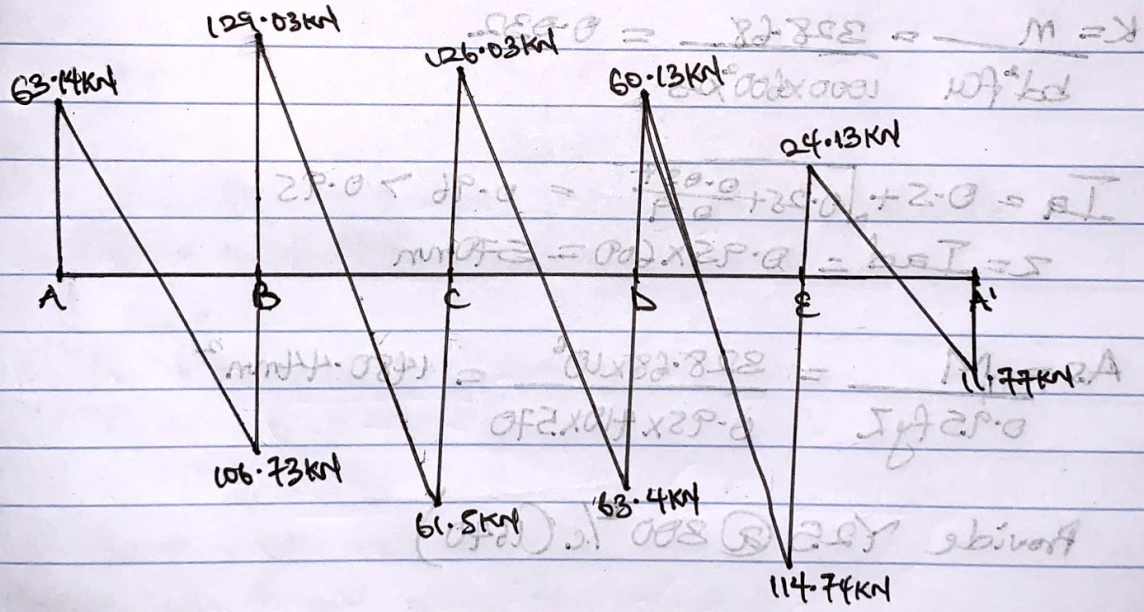

$V_E = \frac{wL}{2} = V_{A'}$

$= \frac{8.24 \times 1.5}{2} = 6.18 KN$

$V_{EA'} = V_E - \left(\frac{M_E + M_{A'}}{L}\right) = 6.18 - \left(\frac{45.47 + 0}{1.50}\right) = 24.13 KN$

$V_{A'E} = (8.24 \times 1.5) - 24.13$

$= 11.77 KN$




2) **Base Design**

$M = 1200 KN$

Strength = $25 - 410 N/mm^2$

$F_b = 150 KN/m^2$

Area of base req = $\frac{N \times 1.1}{\lambda \times F_b}$ , $\lambda = 1.46$

$\frac{1200 \times 1.1}{1.46 \times 150} = 6.027 m^2$

$\sqrt{6.027} = 2.45m \approx 2.5m$

Net pressure, $f_{net} = \frac{N \times 1.1}{B}$

$\frac{1200 \times 1.1}{2.5} = 505.824 \text{ kN/m}$

moment, $M = \frac{f_{net} l^2}{2}$

where $l = \frac{1}{2}(B - h)$ $h$ = depth of base = 660

$L = \frac{1}{2}(2.5 - 0.225) = 1.638 \approx 1.14\text{m}$

$M = \frac{505.824 \times 1.14^2}{2} = 328.68 \text{ kNm}$

$d = h - \text{cover} - \frac{1}{2}\phi$

$= 660 - 50 - 10 = 600\text{mm}$

$K = \frac{M}{bd^2 f_{cu}} = \frac{328.68}{1000 \times 600^2 \times 25} = 0.032$

$I_a = 0.5 + \sqrt{0.25 + \frac{0.032}{0.9}} = 0.96 > 0.95$

$z = I_a d = 0.95 \times 600 = 570\text{mm}$

$A_s = \frac{M}{0.95 f_y z} = \frac{328.68 \times 10^6}{0.95 \times 410 \times 570} = 1480.44\text{mm}^2$

Provide Y25 @ 300 c/c (1640)

Punching Shear

Column size = 225 x 450mm

$f_{cu} - f_y = 25 - 410\text{mm}$

Area footing = $6.027\text{m}^2$

Size of footing = 2500 x 2500

$q$ = Net pressure = 505.824 kN/m

depth = 600

Critical section, $d/2 = 300$

$300 + 800 + 225 = 825mm$

$300 + 300 + 450 = 1050mm$

Shear force $V_n = q_n \times [\text{Area of footing} - (0.3+d)^2)$

$= 505.82(2.5 \times 2.5 - (0.3+0.6)^2)$

$V_n = 2751.68 KN$

Normal shear stress $\tau_v = \frac{V_n}{bd}$

b = Perimeter of critical section.

d = effective span/depth.

$\tau_v = \frac{2751.68 \times 10^3}{((2 \times (825) + 2(1050)) \times 600)}$

$\tau_v = 1.223 N/mm^2$

Permisible shear stress

$\tau_c' = K_s \times \tau_c$

$K_s = (0.5 + \beta_c)$ but not greater than 1

$\beta_c$ = Ratio of shorter to larger side of columns.

$\tau_c = 0.25\sqrt{f_{ck}}$

$K_s = 1$

$\tau_c' = 0.25\sqrt{25} = 1.25 N/mm^2$

$\tau_v = 1.223 N/mm^2$

$\tau_v \leq \tau_c'$

hence depth assumed is OK

Checking for $f_b$ with actual size of footing

Unit weight of concrete = $24 KN/mm^2$

Unit weight of soil = $1.091 \times 10^{-6} KN/mm^2$

Actual pressure footing below

$q = ((1200/2.5 \times 2.5) + (24 \times 0.660) + (1.091 \times 10^{-6} \times 0.64)$

$q = 214.94 KN/m^2$